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Rion Antirion Bridge, Greece – Concept, Construction and Analysis

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Abstract: The Rion Antirion bridge is a large multi span cable-stayed bridge located in Patras, Greece. The bridge has three main spans of 560m and two approach spans of 286m in length. Its design, concept and construction were all affected by the unique environment of the area: high water depths, weak alluvium, high wind speeds and the potential for severe seismic loading led to an innovative and outstanding feat of engineering. This paper presents an overview of the concept, design and construction challenges of the Rion Antirion bridge. This is followed by an dynamic analysis of the bridge including the critical stresses in the bridge and the efficiency with which they are taken when subject to British Standards limit state design as well as considering the effects of temperature and oscillations. The author has provided a subjective view on most aspects and on any possible improvements to the bridge.

Keywords: *Cable-stayed Bridge, Seismic Loading, Pre-Strain, Composite Deck*

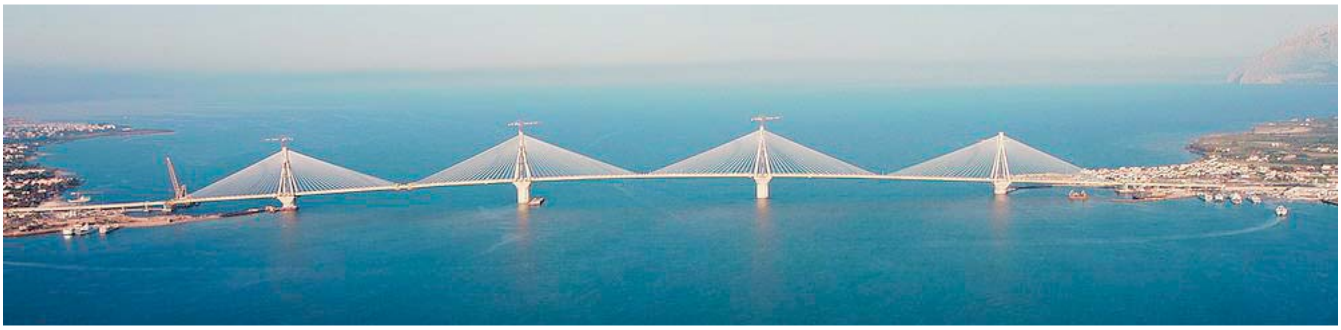

Figure 1: General View

1 Introduction

The Rion-Antirion bridge is a multi-span cable-stayed bridge located in the western end of the Gulf of Corinth in Greece. The bridge links the Peloponnese (Southern Greece) to the mainland as well as uniting two major roads. Currently 10,000 vehicles cross on a daily basis with a crossing time of around 5 minutes; significantly quicker than the pre-existing ferry crossing. As part of the Trans-European Transportation Network, the bridge provides efficient distribution of goods to the region and ports of Italy, thus facilitating commerce with Western Europe. The bridge has improved development and strengthened communications with western Greece, which has suffered from underdevelopment and unemployment.

The vision to span the Gulf of Corinth reaches back over 100 years but due to the environmental restraints in the area the project was not technically feasible until the late 20th century. A Concession contract (i.e. finance, design, build, operate and maintain) was finally awarded in 1993 and financing for the 900 million USD project closed in 1997 allowing construction to get underway. Costs were largely achieved through public funds, private equity and bank loans.

Following seven years of construction Rion Antirion officially opened to traffic in 2004. With a suspended deck 27,20m wide and 2,252m long the bridge is officially the second longest cable stayed bridge in the world (after the Millau viaduct in France). It is widely regarded as a remarkable feat of engineering not least because of its size but particularly due to the harsh environmental restraints that had to be overcome to allow completion of this bridge.

2 Environment

The Gulf of Corinth’s unique environment presented an exceptional combination of difficult physical conditions:

i) Water depths in excess of 65m.
ii) Very weak and deep soil strata consisting of Alluvium to depths of 500m beneath the sea bed.
iii) Significant seismic activity in the area.
iv) Tectonic movement – the two ends of the bridge are founded on different tectonic plates resulting in the Gulf expanding at a rate of 30mm a year.

Considered separately, any of the difficulties listed above would not normally be a major concern. However, the conjunction of all these arduous conditions lead to a number of interesting conceptual issues and resulted in several unique engineering solutions. The development and use of these engineering solutions will be discussed and analysed later in the paper.

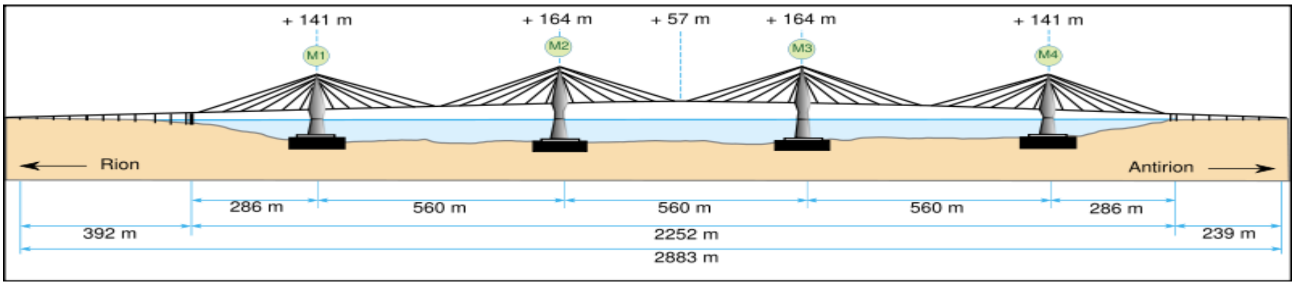

Figure 2: Bridge Elevation & Details

3 Design Criteria

The environmental factors above along with any collision and wind loads dictated to a degree what loads and restraints the bridge had to resist:

1. **EARTHQUAKE –** Bridge designed to withstand the seismic forces and movement corresponding to an earthquake measuring 7.5+ on the Richter scale.
2. **TECTONIC MOVEMENT –**Bridge designed to accommodate possible fault movements of up to 2 metres in all directions, differential displacements between one part of the bridge relative to another as well as a possible inclination in some of the pylons.
3. **SHIP COLLISION –**Pylons built to withstand impact of a 180,000 ton tanker moving at 30km/h.
4. **WIND –**Bridge has a maximum wind tolerance of a 260km/h gust speed.

4 Bridge Concept

Rion Antirion consists of four very large pylons each with a reference span of 560m as well as two approach viaducts either side of the Corinth strait, 392m and 239m respectively. The potential for high soil structure interaction forces with the bridge supports during seismic activity meant that the span length of the bridge had to be adjusted to minimise the number of supports in the strait, particularly in areas of deep water where the soil layers are weakest. These particular restraints may seem to favour the design of a suspension bridge but a major problem with the slope stability on the side of the strait, specifically the potential for liquefaction, precluded this possibility.

4.1 Pylon Design

The large stiff pylons are the main structural elements of Rion Antirion relied on to carry any live load moments (bending). Constructed from reinforced concrete they reach a total height of 160m above sea level. Each pylon consists of four legs, 4 x 4m which converge at the top to provide the necessary rigidity to cope with seismic forces and asymmetrical service loads from adverse distribution of HA and HB traffic loading. This creates a 3-D triangular 'closed' section providing the necessary torsional resistance. Beneath the deck the pylons are embedded into the inverted pyramidal pier head to create a monolithic structure from top to bottom. The huge pyramidal capitals are heavily reinforced and prestressed to allow the structure to transmit the massive forces under seismic loading from the pylon legs to the shafts. The huge octagonal piers which support the pyramidal capital alleviate the wind load on them because of the aerodynamic properties of their shape (although circular piers would have been more efficient still). However, their height/breadth ratio is small increasing its susceptibility to sway and negating any negative aspects of its shape

Creating a continuous pylon proved to be the simplest and most efficient solution (when used in conjunction with a continuous suspended deck – explained later) but was not the only one. Indeed many cable-stayed bridges differ from this and feature a complicated array of sophisticated bearing devices, post tensioned tendons, dampers and more all necessary in a static bridge structure to support the cantilevered sections of deck that span from either side of the pier. However the use a continuous suspended deck that is isolated from the pylons as much as possible still allows the deck to be constructed by cantilevering out from each side of the tower (uses the cables as both permanent and temporary supports) but also improves efficiency and offers far superior resistance to seismic movement.

4.2 Deck Design

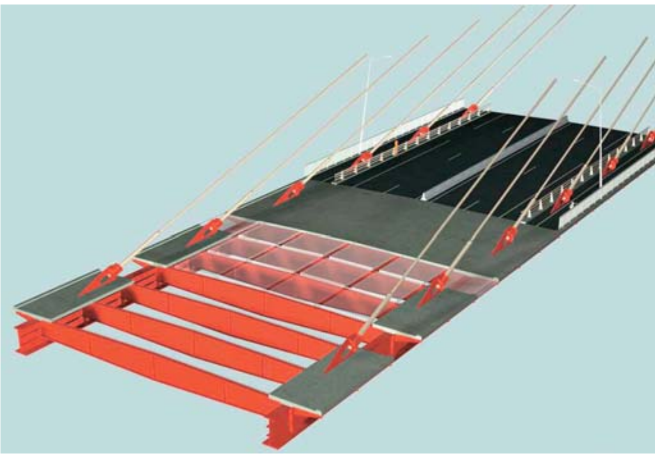

Figure 3: Cut through view of composite deck concept

The relatively thin deck which features two traffic lanes, a safety lane and a pedestrian walkway in both directions is a composite steel-concrete construction and as mentioned is unique in that it is the longest continuous suspended deck in the world. Twin longitudinal steel girders support a 25-30cm pre-cast concrete top slab made up of panels. The girders (shown in red – Fig 4) are 2.2m high and are braced every 4m by transverse cross beams. The use of a composite deck provided the necessary lightness and flexibility to cope with seismic loads. Further advantages of a composite deck allowed the amounts of material to be dictated to ensure the deck carried compression and tension efficiently and also allowed the concrete top slab to be prefabricated and easily placed in the marine environment using the balanced cantilever method. Expansion joints located at each end allow the bridge to accommodate a huge amount of seismic or thermal movement in the longitudinal direction - 2.5m displacements under service conditions and movements of up to 6.0m in an extreme seismic event. In the transverse direction, a series of hydraulic dampers are connected to each pier along with a metallic strut in order to isolate and protect the deck as much as possible as well as dissipate any pendulum movement perpendicular to the bridge span. A further advantage of the suspended deck is that the depth of the deck can be reduced (no large hogging moments over pier bearings) improving its susceptibility to the effects of wind loading. It should be noted that this was an important design factor considering the large span length of the deck and in particular the non-aerodynamic cross section. However reducing the depth of the deck will reduce some of the stiffness inherent in a cable-stayed bridge deck increasing deformations under live loading and the risk of buckling possibly acting as a disadvantage. Clearly a balance was necessary in order to reduce the deck depth without reducing its ability to carry compression loads from the cables or creating an imbalance in the proportions of the structure

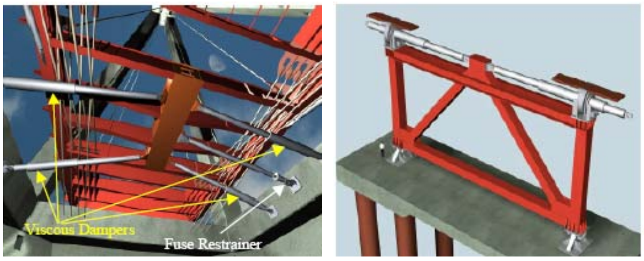

Figure 4: Transverse Dampers underneath deck.

4.3 Cable Stays

The deck is fully suspended by 8 sets of 23 pairs of cables arranged in two inclined planes in a semi fan configuration. The numerous cables also facilitate the use of a slender deck as the cables can be used to transfer bending directly from the deck to the pylons. This direct load path serves to stiffen the bridge considerably and is a major advantage of this type of bridge. The use of a semi fan configuration where the stays converge above the four pylon legs has both structural and aesthetic benefits. Structurally, the greater inclination of the cables in a fan configuration puts less compression into the deck and into the stays reducing the depth of the deck allowing the use of smaller sized cables. Following application of live load the cables will have experienced a degree of stress relaxation requiring pre-straining (shortening) to alleviate this effect and reduce the hogging moment in the deck. This is discussed in detail in 6.2.

4.4 Foundations & Geotechnices

The design of Rion Antirion's foundations was particularly complicated but equally innovative. The two defining design criteria for the foundations was to ensure that the soil had sufficient strength to cope with the soil-structure interaction forces and that the structure was flexible enough to resist significant lateral displacements- both without exceeding performance requirements.

The weak heterogeneous alluvium present beneath the sea bed in the Corinth strait made the initial criteria in particular very difficult to satisfy. Shallow foundations were deemed the most suitable to spread the forces from the pylons to the weak soil below as well as providing lateral stability against any overturning moments. Piles and deep foundation would not have been as efficient at dealing with seismic loading and also faced the risk of shearing at the base when subject to sway during an earthquake.

The foundations take the form of large round concrete substructures 90m high and 65m in diameter.

To improve the bearing capacity of the soil beneath the foundations hollow steel pipes 2m in diameter and 25-30m long were driven in at regular intervals in essence creating a reinforced steel-clay composite. The inclusions increase the shear strength of the soil allowing it to take seismic forces as well as the considerable water pressures generated within the soil during an earthquake. It should be noted that although the inclusions bear a resemblance to pile foundations, they do not behave as such. There is no connection between the pylon rafts and the steel inclusions allowing the pylon bases to move in any direction independent of the steel inclusions. Without this the foundations would not have had sufficient lateral rigidity.

Interestingly, instead of the pylon bases being buried into the seabed they almost 'float' on top a 3m thick gravel layer which covers the steel pipes. This serves to partially separate the reinforced soil from the rigid pylon bases to reduce transfer of shear forces from the reinforce oil to the super-structure. As a result, in the event of an earthquake the pylons are free to slide on the seabed providing additional isolation of the seismic forces. In theory the gravel will also rapidly drain any excess pore-pressures generated when soil is loaded suddenly.

It is obvious that the design of Rion Antirion has been governed by its capability as a whole to withstand the movement and forces generated by a severe earthquake. The way in which the foundations react to this in particular is fundamental but difficult to evaluate without performing detailed analysis on the capacity of the soil and the structure to resist an earthquake. Even still, replicating the movement and associated hazards accurately is challenging. Liquefaction for example is a well known hazard associated with ground movement on weak soils and is likely to be one of the most dangerous side effects of an earthquake. However extensive checks will have been carried out to ensure the reinforced soil when loaded by the large shallow foundations will not suffer from liquefaction.

Secondly, there will have undoubtedly been concerns over the long term settlement of these foundations despite the strengthening of the soil and reduced bearing pressure from the use of large diameter footings. Site investigation and extensive soil testing will be needed to provide parameters for analysis and the development of predictions against which a suitable monitoring scheme can be compared.

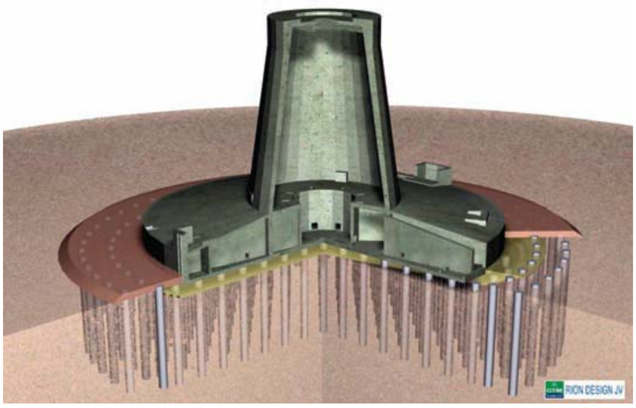

Figure 5: Reinforced soil and foundation concept

5 Aesthetics

What makes a bridge appealing is difficult to quantify. Thankfully no set of rules exist that offer the bridge designer a full-proof method to design an aesthetically appealing bridge. Fritz Leonhard wrote about ten areas of aesthetics which he felt should be considered during bridge design; his views continue to be considered today and will form the basis of a largely objective aesthetic analysis of this bridge.

Simplicity is often the hallmark of an attractive and well designed bridge. Whist Rion Antirion is by no means simple in its structural purpose (largely due to the environment it’s in), from a distance (Fig 1) it conveys an immediate sense of elegance and simplicity of design which masks its underlying structural complexity. This is arguably its most impressive aesthetic feature helping to create a positive first impression. It is both a combination of the simplicity with which the bridge highlights the purpose of its load bearing elements and the refinements to the bridge design which help to create a sense of order and simplicity.

The function of the bridge is clearly shown by the strength of the large pylons and numerous connecting cables which seem to naturally facilitate the use of a slender deck and add a sense of stability. By painting the cables matt white, the designers have made a distinct effort to accentuate the cables so that they appear from a distance almost as a semi-transparent mass. The deck does not look out of proportion with the rest of the bridge as it appears to connect smoothly as the bottom edge of this triangular mass. Perhaps a downside of using so many cables is that the load path from deck to pylon is not quite as obvious as it is in cable-stayed bridges which feature fewer cables. The use of a semi fan shape configuration adds to the order of the bridge by eliminating any potential criss-crossing of the cables. A further refinement of this concept resulted in the cables from each side of the deck converging in a vertical line at the pylon head creating a pleasant tent like space that is well defined and devoid of unpleasant crossings. At the other end the cables connect to the top of the deck in a subtle and discrete manner (Fig 4) that is barely visible from distance and also helps to create a sense of order. The aesthetic function of a bridge is by no means limited solely to the illustration of the load bearing elements; the seismic dampers for example have been deliberately hidden beneath the bridge deck to help to avoid any confusing breaks in continuity and allow the eye to flow smoothly over the bridge.

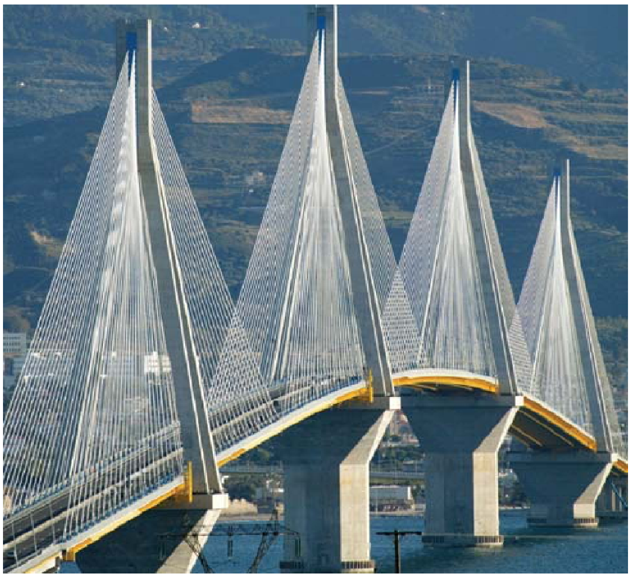

Figure 6: Oblique view of Bridge

As mentioned the octagonal piers support an inverted pyramidal capital to which the 4 slender pylon legs join. The octagonal shape of the piers plays with light and shadow to partially create the pretence of a much slender element as demonstrated to a degree by Fig 7. Hexagonal piers might have improved this effect although possibly at the expense of the feeling of stability and proportion.

It might be argued that the four slender pylon legs look out of proportion when compared to the large solid supporting piers. Although this is a valid point, the design is a refreshing change to the traditional A shaped tower; the four legs match the relative slenderness of the stay cables, allow an unrestricted view across the bridge from an oblique angle and appear as a natural extension of the piers directing your view skyward. Indeed one of the major refinements of the design was to embed the legs into the pier to create a continuous monolithic structure. When viewed in pure elevation (Fig 1) the pylons look particularly simple as the four pylon legs merge into two. Perhaps surprisingly even when viewed from an oblique angle the pylon legs maintain their sense of simplicity which possibly is largely an attribute of their slenderness. Their shape has been particularly dramatised at night where a series of deep blue lights illuminate the pylons from below leaving the top to gradually disappear into the darkness:

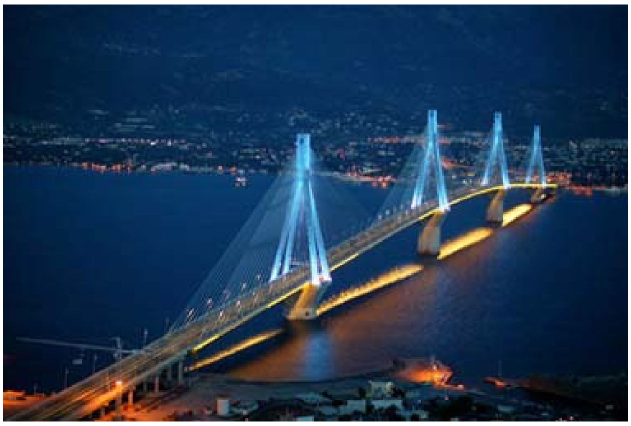

Figure 7: Bridge viewed at night.

Only three surfaces of the pylon legs were lit, each with differing luminous intensity in order to emphasise and highlight their form. Similarly the deck lighting (a thin golden thread along the entire length) has been designed as a superimposition of directed shadows, which provide this line with relief, texture and animation. However, whilst the blue lighting on the pylon legs is subtle and blends to a degree into its background, the yellow lighting is particularly out of place and conflicts with the blue.

The piers of the approach viaduct are much smaller than the main piers and when viewed along the length of the bridge contrast with the bulkiness of the main piers. However, this a minor issue and in reality the approach piers are low and are remarkably well hidden (considering the high clearance of the main span) by the surrounding infrastructure avoiding any clash in proportions. Unfortunately it is the view from the approach spans (Fig 7) that offers the worst view of Rion Antirion. No longer does the design look elegant, clean and crisp but instead the huge piers look particularly bulky, orthogonal and out of proportion with the relatively slender deck. The shape does little to alleviate this aspect. The concrete appears dirty and the yellow strip running along the deck looks particularly out of place and clashes with the colour of the bridge.

Rion Antirion is a bridge with two sides to its aesthetics. Viewed as a whole from a distance and in the sun, it appears particularly elegant in spite of its large dimensions; the slenderness of the deck and pylons helps it to blend into the landscape. The whiteness of the concrete further helps to integrate it as well as accentuating certain aspects. Sadly this is lost when it is viewed from close up where its bulkiness is made starkly apparent. However, despite its underlying complexity, it still manages to portray its function in a simple fashion. The refinements to the design certainly contribute to this defining and hiding certain elements to achieve a sense of order whilst maintaining a degree of complexity and character. Rion Antirion’s legacy may be as a feat of engineering but its aesthetic qualities should not be discounted.

5 Construction

The construction method evolved by considering the bridge design, a study of realistic construction methods, and the environment the bridge is situated in. The methods taken to combat the harsh environment of the strait (dredging, steel inclusions, gravel bed etc) required some exceptional and indeed unique works to be undertaken. As a result some of the latest technology and equipment related to this field was necessary as well as a considerable amount of skill.

5.1 Foundations &Tension Leg Platform

Initially the seabed was dredged at pylon locations before a 90cm thick sand layer was laid on top. Steel inclusions (left protruding 1,5m above the sand) were driven in to the clay to reinforce it. This was covered with a thick layer of rounded river gravel (1.6 – 2.3m) followed by a thin layer of crushed gravel (50cm).

Works were performed out of a 60m long and 40m wide tensioned leg platform tethered to the seabed by movable concrete blocks attached with adjustable chains. This is essentially a large vertically moored floating structure which experiences minimal vertical motion making it particularly good for controlling construction accurately. The marine equipment used to and prepare the seabed and drive the steel inclusions was mounted to the side of the platform on a submersible pontoon attached with steel arms. An extremely long movable steel tube which almost reached to the seafloor guided the equipment and materials to the desired location. This allowed works to be carried out on a 14m wide and 28m long area.

A large barge moved the platform to over 40 different locations over the course of about 5 months using a dynamic positioning system. Precise monitoring of the foundation level was achieved using sonar scanning equipment mounted to the platform ensuring it was within accepted tolerances.

5.2 Pylon Bases

The pylon bases which consist of a footing and a connecting conical shaft were built separately. The footings were cast first in a dry dock 230m long and 100m wide followed by the conical shafts cast in a wet dock with sufficient water depth.

The dry dock allowed the casting of two different sized cellular pylon footings at a time. Two different levels in the dock (Fig 9) provided 12m of water for the leading pylons and 8 m for the pylons nearest land. Each footing was finished by adding the bottom section of the tapering shaft to allow an easier and more accurate connection between the two parts. Once one footing was complete the dock was flooded and the footing town out a few hundred metres to the deep water wet dock (Fig 10).

Here the conical shaft of the main pylon base (anchored with chains) was progressively cast with standard jump forms on top of the footing. To improve ease of construction it was necessary for the footing to maintain a constant height above water by progressively flooding the cells in the base of the shaft. Once the pylon base had been built to the required height it was moved to its pre-prepared bed where it was ballasted and placed at its final location. The base was pre-loaded with water in order to speed up and anticipate expected settlements during pylon shaft and capitol construction. This allowed the exact settlements to be accrued and in particular allowed accurate corrections to be made for differential settlements between separate pylon bases.

Of particular interest is the innovative way in which the dry dock was kept watertight. The first castings were protected using a standard sheet pile dyke which consequently had to be completely removed to allow the dock to be flooded. To avoid the added cost and time of building a new dyke every time the dock was flooded, the smaller pylon was floated forward and sunk by flooding to act effectively as a gate. Temporary steel walls were around the base top slab and sheet piles projecting from its sides sealed the dock allowing it be dewatered and reused.

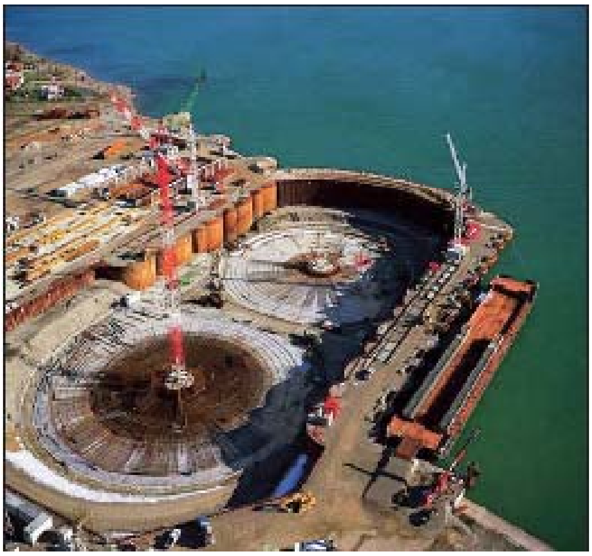

Figure 8: Dry Dock Construction.

5.3 Upper Pylons

The upper sections of the pylons were constructed using a large support barge as a fixed base to provide all the necessary materials and equipment. A smaller roll-on roll-off barge re-supplied the base with truck mixers, reinforcement and more. The large octagonal shafts of the pylons were cast in situ using self climbing formworks. As mentioned, the pylon capitals are vital parts of the structure and required heavy prestressing in order to suitably transfer the forces down through the structure. Their construction was particularly difficult and arduous taking 7 months to complete and requiring a huge amount of concrete, steel and sophisticated equipment. The four pylon legs were cast in 4.8m sections up to the point where they converge. Seismic resistance during construction was provided with heavy temporary bracing. Lastly the steel core of the pylon head was craned into place by a huge floating crane able to reach heights of 170m above sea level.

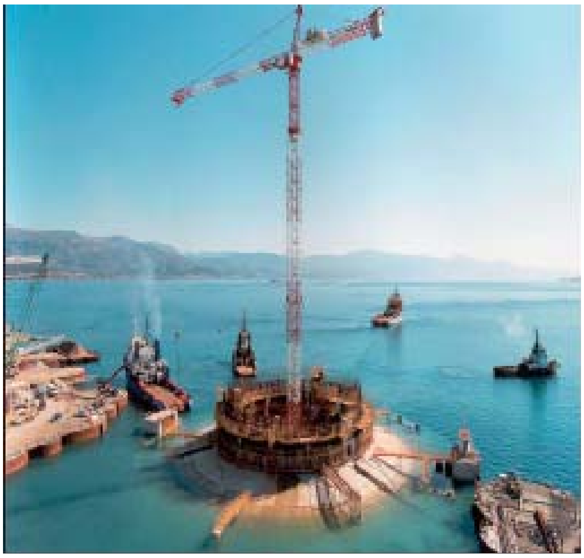

Figure 9: Towing of Pylon Bases.

5.4 Deck

The deck was constructed using the balanced cantilever method – the cable stays were used as both temporary and permanent supports allowing the deck to be progressively cantilevered out from each pylon before the ‘key’ segment was inserted to join the adjacent sections of cantilevered deck. Pre-stressing tendons in the concrete resist the large hogging moments generated principally by the dead load of the cantilever. These construction stresses are usually similar to the permanent stresses once construction is complete and hence the pre-stressing schemes do not have to be especially refined.

The lack of any separate temporary supports is one of the primary reasons why cable-stayed bridges are so cost effective. Furthermore this construction method is particularly suited for long-span bridges where construction activities for the superstructure can be planned at deck level.

Deck elements 12m long including the top concrete slab were prefabricated at a pre-assembly yard and hoisted into position using a floating crane before being bolted to adjoining sections. The deck elements are roughly the same length as the cable spacing which presumably to ensure that the cables were fully capable of supporting the segments during construction without the need for additional expensive pre-stressed tendons. Only a small keying segment was needed to allow enough space for the pre-stressing jack.

6 Dynamic Analysis

Rion Antirion has been designed according to limit state philosophy; i.e. checks in accordance with presumably Greek standards have been performed to ensure the bridge is adequate in ULS (Ultimate Limit State) and SLS (Serviceability Limit State). Certain elements would have required specialist literature, extensive testing or CFD simulation to review the performance of the bridge when loaded, particularly to help review the bridge against the critical design criteria mentioned in section 3. Performing a full structural review of the bridge is beyond the scope of the paper and is perhaps superfluous considering the bridge is still functioning normally. Despite this, a series of simple strength checks have been performed on the deck to ensure it is within acceptable parameters, serviceable and importantly efficient in taking loading.

In order to do this it is first necessary to define the design loads in line with BS5400 that will be required to reassess the bridge. These will essentially be the dead, super-imposed dead, live, wind and temperature loads that the bridge is currently experiencing. Note that the live loading will only consist of the more critical HA loading. All secondary live loads that can occur due to traffic such as centrifugal loading, HB loading, collision and longitudinal loading have not been evaluated as they are not required in the following evaluation. Similarly the effects of creep in the composite deck were also considered minimal.

6.1 Vertical Design Loading

ULS	Unfactored W (kN/m)	ɣF1	ɣF3	kN/m	kN/m^2
WdlɣF1 ɣF3	270.00	1.15(concrete) 1.05 (steel)	1.15(concrete) 1.05 (steel)	338.38	12.53
WsdlɣF1 ɣF3	219.84	1.75	1.75	673.26	24.94
WHAɣF1 ɣF3 (HA Alone)	17.20	1.50	1.25	129.00	4.78
WHAɣF1 ɣF3 (HA&HB)	17.20	1.30	1.10	98.38	3.64

SLS	Unfactored W (kN/m)	ɣF1	ɣF3	kN/m	kN/m^2
WdlɣF1 ɣF3	270.00	1.00	1.00	270.00	10.00
WsdlɣF1 ɣF3	219.84	1.20	1.20	316.57	11.72
WHAɣF1 ɣF3 (HA Alone)	17.20	1.20	1.00	82.56	3.06
WHAɣF1 ɣF3 (HA&HB)	17.20	1.10	1.00	75.68	2.80

6.1.1 Dead Load

The dead load of the suspended deck has been taken as the combined weight of the concrete finish and the supporting steel girders. Values have been conservatively rounded up. The mass of the 2.2m steel girders have been taken as twice that of a UB[1] 1016 x 305 x 487 which weighs 486.6Kg/m

Steel: 2x486. 6= 973.2Kgm/ = 9.55kN/m length.
Concrete (plain): 2300Kg/m^3=22.56kN/m length.

Longitudinal Girders: 9.55 x 2 ~ 20kN/m. **(1)**
Transverse Girders: (9.55 x 27)/4 ~ 65kN/m.
Concrete: 22.56 x 0.3 x 27 ~ 185kN/m.

Total = 270kN/m or 10kN/m^2.

6.1.2 Super-Imposed Dead Load

The super-imposed dead load of the structure consists of the weight of all the materials on the structure that do not serve a structural purpose. A value of 1 kN/m^2 has been taken to account for the services (piping, drainage etc). In addition it has been assumed that a 300m thick layer of asphalt sits atop 100mm of saturated sand fill.

Sand (Saturated) = 2000kg/m^3. **(2)**
Asphalt = 2300kg/m^3.

6.1.3 Live Traffic Load – HA Load

Live traffic loading considered here consists only of full; HA loading across the length of the deck. The effect of the nominal Knife Edge Load usually applied in conjunction with HA loading is minimal and hence has not been used or evaluated. HA loading is a formula loading representing normal traffic in Great Britain. [2] For span lengths in excess of 380m, a notional value of 17.2kN/m is used[3]. This value has been multiplied by the number of notional lanes to get a UDL across the length of the deck.

Notional Lanes = 4 (Carriageway ~12m). **(3)**

6.2 Moment in the Deck

If the deck is considered to be continuous and supported every 10m (roughly where each cable connects to the deck), rough theory would dictate the moment distribution to be as follows (black line):

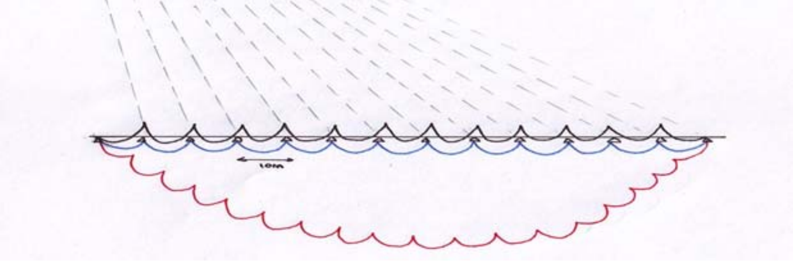

Figure 10: Moment Distribution in deck under UDL.

However, in reality stress relaxation of the steel tendons over time causes the entire deck to sag as a whole moving the above moment distribution downwards (red line). In order to reduce this effect, the cables are post-tensioned (shortened) after relaxing to move the moment diagram back up. The amount of pre-strain can be controlled in effect allowing the moment in the deck to be dictated to whatever is deemed optimum: a major advantage of cable-stayed bridge design. Modern bridge decks tend to be superior in sagging and hence if possible the cables are pre-strained so the deck carries no hogging with minimal sagging (blue line).

In Rion Antirion's case it is very difficult (without access to the correct information) to judge the level of pre-strain in the cables and hence the corresponding moment in the deck. However, a clue as to the level of pre-strain is in the size of the bridge. In long span bridges it is particularly difficult to achieve high levels of pre-strain. A balance is quickly reached whereby any additional tension would require unfeasibly large cables to cope with the extra self weight and sag induced stress. Instead the cables are pre-strained to an acceptable limit and the bridge is designed to cope with the large sagging moments ensuring that the sagging moments do not exceed the plastic moment capacity of the deck. Hence it is a reasonable assumption to say the maximum hogging moment in the deck will be just less than the moment capacity of the deck but far greater than the maximum hogging allowed if the cables were fully pre-strained (black line – Fig 11).

The following check will determine this accuracy of this assumption between the largest span (assuming the deck is pinned at the piers – not strictly accurate)

6.2.1 Plastic Moment Capacity of Deck

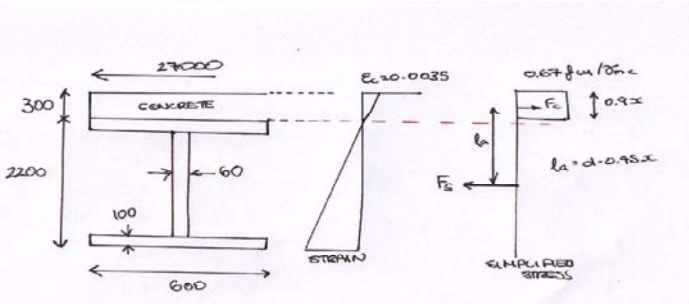

Figure 11: Deck Section in mm with assumed dimensions (not to scale).

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[1] Structural Sections to BS4: Part 1: 1993 and BS EN10056: 1999

[2] 6.1 BS5400

[3] Unfactored HA UDL [Fig. 10 BS5400]
Unfactored KEL [6.2.2 BS5400]

Assume Mild steel (F_y=300N/mm^2) and C30 Concrete.

Assume steel yields so that $\mathbf{F_s = F_c}$:

F_s =Area Steel x F_y/1.15 = 480000 x 300/1.15.
F_s = 192MN.
Equating: F_c = 192x10^6 = 0.45F_{cu} x b x 0.9x F_c=0.45x30x27000x0.9x.
Rearranging: x = 381.7mm. **(4)**

This indicates a neutral axis location very close to the concrete/steel join as would be expected for an efficient design.

Plastic Moment Capacity=F_cxL_a =192 x (1.4-(0.45x0.38) = 235.97MNM **(5)**

6.2.2 Comparison of Moments

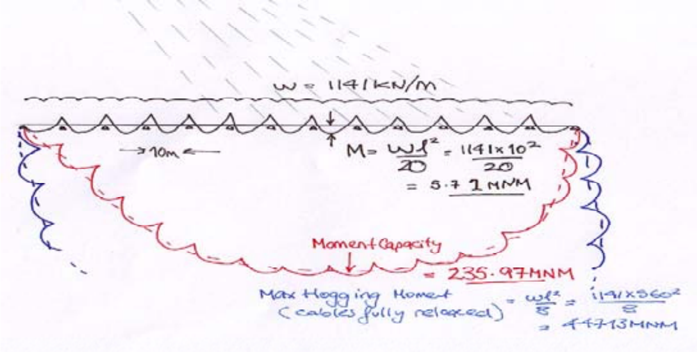

Figure 12: Comparison of Moments.

The plastic moment capacity of the deck gives the best indication of the moment distribution in the deck post relaxation of the cables. It would be inefficient to use a deck with a much greater moment capacity than the expected worst case moment. Hence the maximum hogging moment allowable in the deck will be slightly less than 235.97MNM, far greater than the maximum hogging allowed if the cables were fully prestrained (5.71MNM). This indicates as expected that the bridge has been designed to cope with the sagging of the deck and not pre-strained to a high degree. Although speculative, it is likely that the cables have been pre-strained to carry dl and sdl without relaxing but were intended to relax upon application of live load.

6.3 Compression in the Deck

The inclination of the cables induces a compressive stress in the deck serving to stiffen it. The shallower the inclination of the cable, the greater the induced compressive stress. Each cable contributes a compressive force which builds up to a maximum underneath the pylons. For the following calculation the compressive force in the middle segment has been calculated and multiplied by the number of cables (23) to give a rough indication of the maximum compressive force in the deck.

Each cable approximately supports a 10x13.5m segment of deck weighing 38.66kN/m^{2}x10x13.5 = 5219.1kN.

Approximate inclination of middle cable:
tan^{-1} (100/140) = 35.5°.

Therefore compressive force in the deck
= 5219.1tan (90-35.5) = 7316.9kN. **(6)**

Total Compressive Force = 7316.9 x 23 = 168.3MN.
Compressive Stress = Force/Half C.S.A = 168.3/4.29.
Compressive Stress =39.23N/mm^2.

This stress can then be added to the wind induced bending stress to check the total stress in the section when subject to horizontal wind loading. Although the compressive stress calculated above occurs at the pylons, to be conservative it has been assumed to act across the whole length.

6.4 Wind Loading

Although BS5400 is only applicable to bridges situated in the UK with spans up to 200m it still can be used purposefully here. In reality for bridges of this size and complexity the most critical result of wind loading is how the deck reacts to the wind. For this complex CFD modelling and extensive wind tunnel testing will have been carried out to assess the bridges change in geometry and susceptibility to structural galloping and flutter. Rion Antirion currently suffers from none of these effects and whilst BS5400 does not consider them, it can still be used to evaluate with reasonable accuracy the wind load on the bridge in turn allowing the stresses in the steel to be assessed (6.4.2)

Large span bridges are particularly susceptible to wind loading and many modern deck sections (especially in the UK) are aerodynamic in order to relieve the wind. Rion Antirion's deck is particularly un-aerodynamic in section and hence has had to be designed to cope with the additional wind loads. Presumably this design choice was dictated by the greater cost of an aerodynamic deck. Furthermore, the deck uses deep plate girders which have a propensity to catch the wind more than open section trusses for example. This can lead to the deck swaying beyond acceptable limits causing damage to the deck and also the piers. In order to reduce this effect the deck has been made as shallow as possible (by using many closely spaced cables) partly to reduce its drag coefficient and make it less susceptible to catching the wind. Up to this point the use of an active damping system has been attributed to relieving the effects of seismic loading on the deck. However, its effects are likely to be equally effective when dealing with wind induced pendulum movement.

An important aspect of wind loading is the uplift or vertical downward force created by wind. This is particularly relevant for bridge decks that aren't aerodynamic. However, BS5400 only applies to aerodynamic bridge decks and without specialist literature this nominal force has not been calculated or used in the strength checks in 6.2.

The following check assesses the bridges unloaded compressive stress when loaded with a horizontal wind load in relation to the buckling strength of steel:

$100N/mm^2$. A full check would need to consider the superstructure with live load as well.

6.4.1 Wind Load

Assumptions: Mean Hourly wind speed 30m/s, no funnelling, height above ground level 60m, loaded length 560m and depth (d_1) 3m.

$$\text{Max Wind Gust } V_c = vK_1S_1S_2 = 30 \times 1 \times 1.58 \times 1.36$$
$$V_c = 64.46m/s \tag{7}$$
$$\text{Horizontal Wind Load } P_t = 0.613V_c^2A_1C_d$$
$$P_t = 0.613 \times 64.46^2 \times 560 \times 3 \times 1.1 = 4.71MN = 8.41kN/m$$

6.4.2 Stress and Serviceability of Deck

If the deck is considered simply supported between largest span (not strictly accurate) then the elastic bending stress in the steel σ = My/I

$$\text{Where Max } M = WL^2/8 = 8.41 \times 560^2/8 = 329.67MNM$$
$$\text{Second Moment of Area I of transformed section:}$$
$$I = Ay^2+I_{local} \tag{8}$$
$$I = (600x100x13200^2)x4 + (60x2000x13200^2)x2 + ((100x600^3)/12)x4 + ((2000x60^3)/12)x2 + (45x27000^2)/12$$
$$I = 1.57x10^{14}mm^4$$

N.B/Concrete transformed to steel:
Modular Ratio = $E_{steel}/E_{concrete}$ = 200/30 = 6.67
Concrete Breadth = 0.3m/6.67 = 0.045m

$$\text{So, } \sigma_s = (329.67x109x13500)/1.57x10^{14} = 28.35N/mm^2$$
$$\text{Total Compression in Steel} = 28.35+39.23 = 67.58N/mm^2 \tag{9}$$
$$\text{Midspan Deflection } \delta = 5wl^4/384EI$$
$$\delta = 5x8.41x560000^4/384x200000x1.57x10^{14} = 342.97mm$$

The total compression in the steel is less than the notional $100N/mm^2$ usually taken as the buckling strength of steel illustrating the deck is within acceptable limits for horizontal bending. The maximum compression is relatively close to $100N/mm^2$ as would be expected with an efficient design, especially when considering that if the wind was factored the stress is likely to be closer still. A corresponding transverse deflection of 342.97mm appears reasonable and hence shouldn't result in cracking or any other loss in serviceability. In reality a bridge in such an environment will have been designed so that only exceptional loads (earthquakes, tanker impact) would cause cracking in the most heavily loaded zones.

6.5 Temperature Effects

Temperature fluctuations are a particularly important aspect of bridge engineering. Overall temperature changes will expand/contract the bridge deck longitudinally which unless catered for by expansion joints can result in enormous stresses in the bridge deck. Linear temperature differences across the deck section can create additional bending stresses but has not been reviewed here.

Rion Antirion has two expansion joints located at each end of the deck to cope with the effective temperature changes and resultant expansion/contraction of the bridge deck. The following will calculate the expansion of the deck when subject to an overall temperature change of 25°C to check whether it conforms to the acceptable 2.5m serviceability limit Rion Antirion has been designed to.

$$\Delta L = \Delta T \times L \times \alpha \quad \text{where } \alpha = 12x10^{-6}/°C \text{ for concrete/steel}$$
$$\Delta L = 25 \times 2883000 \times 12x10^{-6} = 0.865m < 2.5m \tag{10}$$

This is considerably smaller than the serviceability limit of 2.5m suggesting that in reality the bridge has the potential to experience much greater expansion/contraction under service conditions than calculated, probably because an overall temperature change of 25°C is slightly small considering the location. If the expansion joints were clogged, this change in length would manifest itself as a compressive force in the deck (assuming a rise in temp) which has the potential to cause buckling of the steel girders:

$$\sigma_c = E\mathcal{E} = 200000 \times 0.865/2883 \tag{11}$$
$$\sigma_c = 60 N/mm^2$$

Although not high enough to cause buckling of the steel girders when considered alone, it is a considerable stress. When considered in conjunction with other load induced compression such as the maximum compressive stress in the deck from the cables ($39.23N/mm^2$) it is apparent that this is more than capable of causing failure and illustrates the importance of ensuring that the expansion joints are fully serviceable at all times.

6.7 Natural Frequency

In order to analyse the dynamic phenomena associated with Rion Antirion, in particular the vibrations associated with seismic loading, a prior knowledge of the frequencies and vibration modes of the structure was required. The vibrations due to wind and traffic may not have the same potential to damage the structure as seismic vibrations but can still be a inconvenience to users. These physiological effects are often deemed initially acceptable if the natural frequency of the bridge is between 5 and 75Hz. It should be noted that this design check encompasses a very wide range of frequencies and utilises simple calculations that are not accurate. However, they are still useful to provide an initial evaluation.

Fundamental frequency evaluated between two cable supports (10m separation), deck section transformed to steel:

$$\omega = (\beta_n l)^2 \sqrt{\left(\frac{EI}{ml^4}\right)} = Natural\ Frequency$$

$$I = A\bar{y} + I_{local} = 6.39 \times 10^{11} mm^4 \text{ –calculated similar to 6.4.2 but with deck horizontal} \tag{12}$$

$$\text{Density of steel} = 7850kg/m^3$$
$$\text{Area Section} = 480000+27000x45 = 12630000mm^2$$
$$\text{Mass} = 7850x10^{-9}x12630000 = 99.15kg/mm$$

Therefore, assuming clamped-clamped situation and checking fundamental frequency (mode 1):

$$\omega_1 = (4.73)^2 \sqrt{\left(\frac{200000 \times 6.39 \times 10^{11}}{99.15 \times 10000^4}\right)} = 8.03\text{Hz} > 5\text{Hz}$$

Although this indicates an acceptable natural frequency of the deck, it is not accurate. In reality the natural frequency would be lower (but still acceptable) as the deck is effectively pinned between cables and not fixed.

7 Durability

All modern major structures are designed with future maintenance and durability in mind. Steel and concrete structures like Rion Antirion are no different and require proper maintenance to ensure serviceability and longevity. Concrete is particularly susceptible to cracking which can reduce its structural integrity and expose pre-stressing tendons to corrosion. This can be limited through good design and use of correct concrete mixes. It is certain that these have been adhered to in the design and construction of Rion Antirion, for instance, modern prefabrication methods allowed the deck to be constructed extremely accurately off site. In addition, the concrete finish will have helped to seal any microscopic cracks present in the concrete as well as waterproofing it to a degree to prevent corrosion of the steel tendons. The steel girders and cables are equally susceptible to deterioration (particularly rust due to the saline environment) and again efforts have been made to increase the lifespan of these elements. The steel cables are sheathed in a robust polyethylene that contains a rust inhibiting agent. Similarly the structural girders have been painted with a modern protective coating system for largely the same reasons.

The majority of the structural elements are visible and accessible so any signs of deterioration are readily apparent without the need for extensive investigations. The provision of permanent access platforms allows any necessary repairs to be swiftly and easily addressed.

The aggressive environment Rion Antirion is situated in requires special attention to ensure the durability of the bridge. Although the bridge has been designed to cope with different levels of earthquake loading, it will not be possible to prove that the design assumptions have been satisfied until a real earthquake strikes the area. To maintain the bridge's earthquake readiness necessitates that all the elements of the bridge are functional highlighting the importance of routine maintenance and checks on the structure. The bridge has been designed to be fully functional after small tremors but to require inspection and minor repair after a larger earthquake. In an extreme earthquake event, the bridge will have been designed to survive without collapse, although it may be expected to suffer some damage and be out of commission for a considerable time while repairs are made. Earthquake loading will provide the biggest test of the bridge's elements and durability.

8 Future Changes

Although constructed only recently it is important to consider the scope for future changes to the bridge. As with other major crossings around the world, once built a new bridge like Rion Antirion is likely to attract additional traffic. Whether this traffic increase has been accurately predicted by the Greek Government and accommodated in the design will be a test of time. Nevertheless, a number of future changes might be necessary. At an operational level, it is probably that new electronic systems will be required as bridge controls and toll booths are upgraded. Monitoring systems on the bridge are used to provide information on the performance of the bridge under all conditions as part of the Operating Company's bridge management system. Replacement of key components are necessary as part of the through-life management. Maintenance operations are regularly programmed, including painting and corrosion protection. Cables may also need to be replaced. Changes are certainly possible in the approach roads leading to the bridge and the traffic management system. However the bridge has not been designed to accommodate major changes to the structural design, such as bridge widening. The shape of the pylons together with the positioning of the dampers makes it unfeasible to increase the width of the deck even if the pylons and foundations could be justified for additional load. Fundamentally, the design of Rion Antirion has been carefully optimised and any major structural modification would be extremely difficult.

9 Acknowledgment

-Tim Ibell –Bridges Lecturer University of Bath
-University of Bath Library

9 Conclusion

1.Rion Antirion is a remarkable feat of engineering and design. It has overcome its harsh environment through a number of innovative and unique solutions both in its design and construction to make a long lasting dream a reality.
2. The design of the bridge has been reviewed against British Standards. These calculations have confirmed a number of major aspects of the bridge design.
3. The bridge is expected to continue to provide service as designed throughout its design life.

10 References

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